AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CLXXXIV.

(Vol. VIII.—September, 1879.)

THE OCEAN PIER AT CONEY ISLAND.

By Charles Macdonald, C. E., Member of the Society.

Presented at the Eleventh Annual Convention, June 17th, 1879.

Coney Island is a recent sand bar formation pertaining to the town of Gravesend, at the southwestern corner of Long Island (see Plate XLII), and forming the eastern boundary of the outer bay of New York.

Until within the last three or four years the exceptional advantages afforded by this island for surf bathing and direct access to the cooling atmosphere of the Atlantic were allowed to remain undeveloped and almost unknown, with the exception of a small strip at the west end occupied by Norton & Murray's Hotel, which was reached by boat from a landing in Gravesend Bay.

In the year 1877 the Manhattan Beach Hotel, at the east end of the island, was opened to the public as a place of summer resort, and the extraordinary success attending this enterprise, which included also a railroad to Bay Ridge, stimulated similar investments in succeeding

years, until at this present there is scarcely a foot of beach unoccupied, and the number of daily visitors is to be counted by tens of thousands.

Although the facilities for communicating with the island have kept pace with the rapidly increasing traffic, they all involve more or less railroad transit, which, when considered in connection with the heat and dust of midsummer, and the necessary steamer transfer for all except Brooklyn passengers, too often becomes a fruitful source of inconvenience and delay.

The iron pier which forms the subject of the following paper was built with a view of relieving this difficulty, by affording means of landing directly upon the beach from steamers connecting with all points in the harbor, and at the same time presenting attractions as a promenade and bathing pavilion.

Its construction, involving as it does the application of the water jet as a means of sinking piles, is believed to possess some features of professional interest.

The water jet as a method of sinking piles through sand has been extensively used both in this country and abroad for so many years, and its efficiency and economy are so fully recognized, as to render a general description of the system quite unnecessary.

L. Y. Schermerhorn, Assistant Civil Engineer U. S. A., Member of the Society, has prepared a table "showing application of the water jet to the driving of piles," from which it would appear that as early as the year 1852 piles were sunk by water jet in the foundations for a wharf and warehouse at Decrow's Point, Matagorda Bay, Texas.

In 1854 hollow cast iron pipes were sunk "by jet passing through and discharging at bottom," in the construction of the Pungateague Bay Lighthouse, Chesapeake Bay, the work being in charge of C. Pontez, Assistant Engineer, under Major H. Bache, U. S. A.

In the construction of the Levan and Kent viaducts, England, in 1856, James Brunlees, used "hollow cast iron piles with jet forced through pile and delivered below point, also wooden piles by water jet passing along side of pile."

In 1862, 5 000 wooden piles were sunk in connection with naval operations in Mobile Bay by means of a "jet through detached tube discharging below point of pile."

At Kansas City, 1867 to 1869, O. Chanute, Member of the Society, sank wooden piles by jet, as also the caisson, Pier No. 4, for the bridge crossing the Missouri river at that point.

In England and on the Continent some of the most interesting applications of the process have occurred in the construction of ocean piers for purposes of promenade and embarkation, while that described in the present paper, built for the Ocean Navigation and Pier Company of New York, and the pier now being constructed at Long Branch, are the first instances of similar work in this country.

The ocean pier at Coney Island starts from high water mark at a point about 500 feet west of the Observatory (see Plate XLII), and extends 1 000 feet in a southerly direction to a point at which a depth of 16 feet of water was secured at high water.

Previous to the erection of the buildings it presented the appearance indicated in the parallel perspective, Fig. 2, Plate XLIII.

At the shore end it is 120 feet wide between centres of outer piles, and 120 feet long; thence to the midsection, a distance of 320 feet, the width is 50 feet; at this section the width is 83 feet, for a length of 100 feet; from thence to the pier head the width is again 50 feet, for 320 feet in length, and the pier head is 100 feet wide by 140 feet long. A double row of wooden guard piles extends around three sides of the pier head, and encloses a triangular space in front, which is floored over and serves as a landing stage. There are two floors throughout, with the exception of the shore end, where the upper one is omitted, except as a part of the buildings subsequently erected. The level of the lower floor is 12 feet above high water, and of the upper floor 24 feet. At the junction of the pier head and main stem there are two flights of stairs ; and at this point the lower floor is practically divided into two—the outer end being used for purposes of embarkation, and all of the remainder, back to the shore end, devoted to bathing houses. Communication with the water is obtained by stairways directly from the lower floor, and by a passageway carried around the west side of the shore building, to high water mark. The main entrance to the pier at this end is reached by a triple flight of stairs at the centre.

After that portion of the structure above referred to was placed under contract, the directors of the company determined upon the erection of extensive buildings thereon, for which the designs of Messrs. Gambrill & Ficken, architects, of New York, were accepted. A general idea of the appearance of this combination may be had from Fig. 1, Plate XLIII, which is taken from the original water color drawing submitted by the architects. At the shore end there is a restaurant on the level of the upper

floor, with kitchen, laundry, and offices below. The promenade deck is covered by a light roof spanning the entire width of 50 feet. At the mid-section, refreshment rooms occupy the enlargement, so as to leave the promenade unobstructed, while over head provision is made for aerial sleeping apartments, surmounted by a clock tower reaching a height of 80 feet above the water. The entire pier head is enclosed as a music hall, with refreshment rooms at the corners and elevated observation galleries on the ocean face.

It is questionable whether buildings presenting any considerable surface to the action of the wind, in such an exposed position and upon such a foundation, may be considered as entirely safe. The present structures have thus far shown no evidence of weakness under the ordinary summer storms; it remains to be seen how they will weather the more violent tempests of other seasons of the year. If the experiment proves successful it will, doubtless, lead to many novel additions to the architecture of that attractive little island.

The wrought iron piles used in this structure are lap-welded tubes, $8\frac{5}{8}$ inches outside diameter, and half an inch thick, except at the shore end where they are of the standard thickness $\frac{3}{8}$ of an inch. As delivered from the works of Messrs. Morris, Tasker & Co., these tubes were furnished with a standard thread at each end and a coupling 5 inches long, by which they were combined at the site, forming the required length. They were also treated, while hot, with a preparation of coal tar and oil inside and out, with a view of preserving the iron from oxidation. This, however, has not proved to be satisfactory, as in many instances the coating has already been washed off. In all such cases it would be desirable to use a marine varnish of some brand which has been thoroughly tested.

All piles from the point where the two floors begin, have their upper sections 12 feet long, so as to bring the coupling directly under the casting supporting the lower floor; the remaining sections do not exceed 20 feet.

In the direction of the length of the pier, the distance between each row is 20 feet.

At the shore end there are eight piles in a row; the four center ones being 16 feet 8 inches, and the outer piles, on either side, 17 feet 6 inches from center to center.

Throughout the main stem there are four piles in a row, 16 feet 8 inches apart.

The mid section has six piles in a row, 16 feet 8 inches apart, while at the pier-head there are eight piles in a row; the two on either side the main stem being 12 feet 6 inches between centres. (See Fig. 2, Plate XLIII.)

The disks, which form a supporting base for the piles, are of cast iron, 2 feet in diameter and 9 inches deep. On the upper side is a socket in which the pile rests, and is secured by four set screws; underneath there is a conical projection, strengthened by ribs, through which an opening 2 inches in diameter is left for the water jet.

On the tops of the piles are cast-iron capitals, secured to them by four set screws and fitted to receive the wrought-iron beams which form the support of the floor; they are also furnished with cylindrical lugs for connecting a system of diagonal brace-rods 1½ inches in diameter.

The beams in the lower floor are supported upon castings clamped around the piles directly above a coupling. These castings have similar lugs for brace-rod connections.

The floor beams are laid lengthways of the pier, and are secured to the castings by ⅞ inch bolts through the lower flanges.

Throughout the upper floor, and in both floors at the pier-head, these beams are 15 inches deep and 150 pounds per yard. Under the bathing houses they are 10 inches deep and 105 pounds to the yard; and at the shore end 12 inches deep and 125 pounds per yard.

They are connected transversely at each row of piles by 6-inch beams, with angle-iron lugs, which are bolted directly through the webs of the main beams; where there are two floors the lower struts are 3 by 5 inches T iron, bolted to the clamp castings. Suitable provision is made for changes in length due to the temperature, by slotting the bolt holes.

As an additional precaution against the possible effect of wind upon the buildings at the mid section and pier-head, a system of vertical bracing in both directions was subsequently introduced between the lower floor and high tide. This consists of 1½-inch rods, connected to the piles by cast-iron rings--which are held in place by tap-bolts—and horizontal struts attached to the same rings at the level of high water. These struts are made up of two 6-inch channel-bars, latticed in the form of an] [section, similar to the lateral struts used in bridges; they necessarily present a considerable surface resistance to the force of the breakers and may prove to be of doubtful utility. Their action in severe weather will be watched with interest.

The timber flooring consists of 3 by 12 inches Georgia pine joist, spaced two feet apart, upon which 3 by 6 inches planking is laid for the upper floor, and 2 by 6 inches for the lower, the whole being suitably spiked and calked.

Owing to the very limited time allowed for the completion of the structure, it was impossible to make suitable soundings and borings in order to determine the proper length of piles; but, from the imperfect data at hand, it was decided to sink the first row 10 feet in the sand and gradually increase the depth to 15 feet at the outer end, where it was expected that the water would be 14 feet at high water. Very soon after work was commenced, it was found that this arrangement could not be adhered to, owing to the irregular slope of the bottom; accordingly about 1 000 additional feet of tubing were ordered, and the piles were made up to lengths which were frequently regulated by the sections of tubing on hand rather than by a strict adherence to the original plan.

At the pier head the water was actually found to be 16 feet deep, and the piles driven in this section were generally 57 feet long, giving a depth of 17 feet in the sand.

In the operation of sinking the piles the water jet was applied through a pipe 1½ inches in diameter, extending down through the pile and disk, and about 2 inches beyond. This pipe was held firmly in position by wooden blocks driven in at the pile head, and connected with the pump by a rubber hose. The water pressure was thus transmitted directly to the sand at the foot of the pile, with the effect, as is well known, of transforming it into a semi-liquid mass, through which the pile settles easily to the required depth.

All piles in the shore section were driven by a machine very similar to the ordinary land pile driver, which could be rolled on the sand as the tide permitted; the pile was kept vertical in its descent by a recess in the machine at its base, while the upper end was clamped to a guide block moving freely in the leaders.

After the completion of the shore end a scaffold was built at a level with the upper deck, upon which was erected the travelling derrick, represented in parallel perspective, Plate XLIV, in which it will be observed that there are four pairs of leaders, sustained upon a framework projecting beyond the points of support, and at right angles to the axis of the pier. These leaders are spaced 16 feet 8 inches apart to correspond with the piles in each row, and the length of the overhang is equivalent to the distance between the rows.

The projecting portion is sustained by four diagonal truss rods passing over upright posts directly over the outer row of wheels, and secured at the rear of the platform.

The entire weight is carried upon two rows of grooved wheels, four in each row, and so arranged as to lead in the line of piles.

At the foot of the overhang, which is 6 feet above high water, recesses are left between the leaders for the guidance of the pile in its descent, the upper end being fastened to a block moving between the leaders.

All the piles in the main stem were driven by the aid of this machine; and it is sufficient to say that its efficacy was fully demonstrated by the accuracy and economy of the work done, and the remarkable rapidity with which it was accomplished.

The outer piles at the end section were driven by the aid of booms rigged on the sides of the main derrick, and projecting guides attached to those already driven.

After the last of the four piles had been placed, at the ocean end, this derrick was dismantled, and the leaders placed upon a frame extending 25 feet on either side, in the proper position for driving the extra rows to complete the pier head, as the machine rolled backward.

Plate XLV, which is reproduced from a photograph taken from the ocean end, represents this modification. On the left a pile is being swung into position for sinking; the small tube extending beyond the end is the 1½ inch pipe used for transmitting the water jet, as before described. In the foreground the connection of the beams and bracing with the piles is shown, and on the right a beam is being placed in position upon piles already sunk. Three of the cast-iron disks are to be seen on the centre platform, one of them already fastened to the end of a pile ready for sinking. Directly over the middle of the frame is a lamp, used for the electric light, to be referred to hereafter.

Plate XLVI represents a photograph taken from the rear of the derrick after the pile shown in Plate XLV had been sunk to the proper depth; this is indicated by a spirit level and straight-edge shown in position, as also the line of hose and its connection with the pile. As long as the jet was kept in motion the pile was easily raised or lowered to the required level, after which the fall lines connected with the pile were made fast at the hoisting engine and the pump stopped, when a very few minutes sufficed to settle the sand back into its place, and the pile became a fixture upon which the iron work could be placed without producing an appreciable settlement.

When no serious obstructions were met with, the time expended in driving a row of piles with the original derrick, placing the iron beams and temporary track and rolling out for the next row, was six hours during the day time, and eight hours on the night shift.

For the first 200 or 300 feet from the shore the piles went down quite rapidly and at a uniform rate, but as the work progressed, and a greater depth became necessary it frequently happened that the pile would bring up on some tenacious material which was assumed to be clay, and through which the water jet, unaided, could not be made to force a passage. In such cases it was found that by raising the pile about six inches and allowing it to drop suddenly, with the jet still in operation, and repeating as rapidly as possible, the obstruction was finally overcome; although in some instances five or six hours were consumed in sinking as many feet. It was not possible to determine the character of these deposits from actual samples recovered, but from the action of the pile in passing through, they were assumed to be clay pockets. These deposits did not appear to extend in layers over any distance, as the experience with one pile would be no criterion for the next; and they varied in thickness from six inches to three feet.

The pumping plant consisted of a Worthington pump with a 12-inch steam cylinder 8½-inch stroke, and a water cylinder 7½ inches in diameter. Also a No. 6 Cameron pump with a steam cylinder 10 inches in diameter, 9-inch stroke, and a water cylinder 6 inches in diameter. The suction hose was 4 inches in diameter, and the discharge hose, which was of four-ply gum, was 3 inches. The boiler was upright, 42 inches in diameter, 8 feet high, and containing 62 tubes 2 inches in diameter.

At first the pump and boiler were placed at the extreme shore end, on a platform 5 feet above high water; from which position all the piles to the end of the mid section were driven. The plant was then moved out upon the lower deck about 400 feet, bringing the pump 14 feet above high water. This arrangement did not give satisfactory results, as the force of the jet seemed to be very much reduced, and the piles occupied more time in sinking; accordingly the pumps were lowered upon a platform suspended four feet above high water, and no further trouble was experienced.

The Worthington pump, although a very old one, did good service. The Cameron also proved very satisfactory, with the one exception of the gum rings, which are inserted in the composition valves under

pressure. These rings wear out in some cases quite rapidly, and cannot be repaired on the work. Where such pumps are to be used at any distance from the factory, a number of extra valves should be provided. An abundance of steam was supplied by the boiler after the exhaust had been turned into the smoke stack and soft coal used as fuel, of which an average of about 160 pounds were consumed in sinking each pile.

With the power above described, it was found that piles could be driven in clear sand at the rate of 3 feet per minute to a depth of 12 feet; after that the rate of progress gradually diminished until at 18 feet a limit was reached beyond which it was not practicable to go without considerable loss of time. It is probable that with a larger pump, and possibly an improved method of applying the jet, better results can be obtained.

A few disks, 2½ feet in diameter, were tried, but much more time was required in sinking them. One 3-feet disk caused a delay of six hours in sinking 15 feet through very uniform sand.

It is to be regreted that the time allotted for the completion of this work, in order to make it available for the summer business, prevented the possibility of testing the merits of other methods of applying the water jet, and of determining the proper size of nozzle in proportion to the power used. In one of the last piles driven the experiment was made of forcing water through the pile itself instead of through an inner tube of small diameter; this was accomplished by clamping a cast-iron cap, with a 3-inch union attached to the top of the pile and connecting the hose directly with it. The joint at the junction of the disk and pile was also made water-tight, so that the water from the pump was forced through the 2-inch opening in the centre.

In this case the pile sank much more rapidly than before, and, although the pump was being driven at full speed, the hose frequently collapsed, which would seem to indicate that the resistance to the movement of the water had been materially reduced.

Upon one or two occasions it became necessary to raise piles which, by mistake, had been driven to a greater depth than required, and allowed to become firmly settled. This was accomplished by passing a jet down around the outside of the pile till the disk was reached, when the pile was easily raised by the blocks attached to the hoisting engine, and finally secured at the proper level.

The limit of safe loading for piles of this character has been assumed to be five tons per square foot of disk ; but of course this must be considered as dependent upon the depth of the pile in the sand, and possibly also the depth of water and compactness of the sand. Owing to the unequal distribution of the weight of the buildings, many of these piles sustain a load due to the structure alone, of 40 000 pounds, or $6\frac{3}{10}$ tons per square foot of disk ; to this must be added the weight of moving masses of people, and exceptional local loads, which have in some cases increased the pressure upon the disk to eight tons per square foot without causing any settlement which may be detected by the eye. This extreme practice is not recommended, however, where absolute rigidity of foundation is required.

The wooden piles around the pier head were also driven by a water jet; the $1\frac{1}{2}$-inch tube used for the iron piles, being lashed to the side of the wooden pile at the top, and held in place at the bottom by three cleats nailed on the sides, forming an opening 2 inches square, through which the tube was easily drawn after the pile was sunk.

It was not found necessary to curve the tube so as to bring the jets under the centre of the piles. They did not exhibit any tendency to work over to the side on which the tube was secured, and were easily controlled in the descent by handspikes from a temporary staging.

As these piles were of pin oak, and consequently much heavier than water, they were settled to a depth of 12 feet without the additional aid of a hammer; but much time would be saved by the use of a regular pile driver *in connection with the water jet* where lighter piles were used or greater depths required.

The most difficult feature of the work was the short time allowed for its completion, in many cases compelling the resort to expedients which might have been improved upon had there been opportunity.

The contract was signed on the 5th of April last, requiring the completion of the pier, exclusive of buildings, by the 5th of June. On the 22d of April the first pile was sunk, and work was continued, with some interruption from high tides, until May 1st, when the shore section was completed. Four days were then expended in erecting the traveling derrick, Plate XLIV.

On May 6th, pile driving was resumed, and on the 28th the last row on the ocean end was finished.

May 29th and 30th were taken up in altering the derrick (Plate XLV),

and on the 4th of June the pierhead was completed, making 286 piles placed, with all the iron beams and connections, in 38 working days. From the 15th to the 28th of May, work was prosecuted day and night, light being supplied by two electric lamps from the Dynamo-Electric Light Company, of Newark, N. J. So far as the light itself was concerned the results were very satisfactory; with one lamp attached to the derrick and one at the shore end, no difficulty was experienced in working a full force of men. The power required to drive the machines, however, proved to be much greater than was represented. A 5-horse power hoisting engine, which was assumed to be sufficient for two lights was, in reality, only capable of driving one machine without being over taxed; and it became abundantly evident that the question of cost of fuel was a very important element in connection with electric illumination.

The pier was opened for business July 1st, and within the next sixty days upwards of 250 000 passengers had been registered, the largest number in any one day being 22 600.

The steamers *Columbia* and *Grand Republic*, rated at 1 800 and 1 900 tons respectively, make regular landings, thus far without injury to the staging. It is proper to state, however, that this portion of the structure is necessarily of a temporary character, and liable to require frequent repairs.

With the exception of the buildings the work was designed and executed by the Delaware Bridge Company, under the supervision of Messrs. Maclay and Davies, Engineers for the Ocean Navigation and Pier Company of New York.

The buildings were constructed upon the design, and under the direction of Messrs. Gamrbill and Ficken, architects, N.Y., to whose skill and originality of conception is largely due the fact that the iron pier at Coney Island is to-day one of the most attractive places on the coast.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CLXXXV.

Vol. VIII.—September, 1879.

STABILITY OF STONE STRUCTURES.

By WILLIAM H. SEARLES, C. E., Member of the Society.

PRESENTED AT THE ELEVENTH ANNUAL CONVENTION, JUNE 17th, 1879.

Structures in stone, wherever built or of whatever design hitherto, depend upon their weight for their stability. Whether they be the rough monoliths of the Druids at Stonehenge, or the monumental obelisks of Egypt, or the more elaborate and graceful columns of Greece, in every case the force of gravity has ever been the only force which gives stability. These pillar-like structures, however, have not been designed to resist any external forces which could overthrow them, except the force of the wind. They either carry no load, as in monumental designs, or, as in the columns of a Grecian portico, they carry only the static load of an entablature, which by its weight as well as its connection with the mass of the building, really adds to the stability of the supporting columns. The height of Grecian columns varies from 6 diameters in the Doric, to 10 or 11 diameters in the Corinthian, while the ancient obelisks are found to vary in the height of shaft between the limits of 9 and 11 diameters.

But in any stone structure designed to resist applied forces which are not vertical, we shall find the relative height much diminished. A retaining wall of first-class masonry against dry sand, can have a height of only three times its thickness, if without surcharge, or of only twice its

thickness with a surcharge equal to half its height. A masonry dam of dressed granite cannot have a height greater than from once and a half to twice the thickness at the base. It is quite evident from these proportions, that the material is massed together for the sake of its weight, so that it may act as a counterpoise against the external pressures, while the strength of the stone hardly enters into the consideration, because it is evidently so greatly in excess of any possible requirements.

Lighthouses exposed to the force of the waves are less than three diameters high to the top of the masonry.

Smeaton's Eddystone lighthouse, on a base of 26 ft., is 68 ft. high, or 2.61 diameters.

The new Eddystone, now building, on a base of 35'.5, will be 116 ft., or 3.25 diameters.

The Bellrock lighthouse, on a base of 42', is 115 ft. high, or 2.75 diameters high.

The Wolf Rock lighthouse on a base of 41' 8'', is 116' 4¾'' high, or 2.80 diameters.

The Minot's Ledge lighthouse, on a base of 30 ft., is 80 ft. high, or 2.67 diameters.

The Spectacle Reef lighthouse, on a base of 32 ft. is 93 high, or 2.91 diameters.

These structures are most carefully constructed as to the bond between stones of the same course by elaborate dovetailing. In some instances the beds of the outer stones are also dovetailed, while dowelling is common in all of them. In the Wolf Rock, and some other lighthouses of the same class, the first and second courses are bolted to the rock with bolts split and wedged at each end, the holes being made slightly conical. But it is admitted by the builders that all these devices are "chiefly useful in the early stages of the progress of a work, when it is exposed to storms, and before the superstructure is raised to such an height as to prevent seas from breaking right over it." Undoubtedly the dovetailing of the beds adds some resistance to an extraneous force which is about to tear the courses apart—so does the adhesion of the cement—but any stability thus gained is due to the tensile strength of the material, a kind of strength in which stone is comparatively deficient. It is stated by Mr. Douglass (Proc. Inst. C. E., March, 1870) that when "blocks of granite are put together in this manner with Portland cement it is found that the work is so homogeneous as to be as nearly *as possible* equal in

strength to solid granite." When it is remembered that the inclined faces of the dovetail amount only to two bands an inch wide in plan by three inches deep, running across the bed of a stone about four feet wide, it will be seen that a very small part of the stone is prepared to resist tensile strain, and the adhesive strength of all the rest of the bed must be due to the cement. It is customary to design important structures in stone, so that they shall be sufficiently stable against the assumed assailing forces, without reference to adhesion of cement or tensile strength of stone. Hence the necessity for large quantities of stone. The lighthouses mentioned are built solid for about one-quarter of their height, and with very heavy walls for the remainder. The new Eddystone lighthouse, now building, will contain 69,100 cubic feet of stone, and is estimated to cost $390,000.

The ordinary bridge pier has a height of from 4 to 7 times the base. The proposed piers of the Poughkeepsie bridge show in the drawings a height 6¾ times the base at the neat lines, the height being about 160 feet. In general no thrust comes upon a pier from the action of the truss, but only from such impulses as may be communicated through the truss, as a whole, by the sudden starting or stopping of trains on the bridge, or by the action of the wind. Whatever these impulses amount to, they are communicated to the bridge seat and transmitted by the pier to the foundations. It is against these strains that the pier is really proportioned, since a small percentage of the section of the pier in plan would be sufficient to carry all the static load, although it would not have the requisite stability against thrust. The large bulk of stone thus demanded to carry comparatively small loads, the consequent cost, and in some cases the difficulty of obtaining suitable foundations for mass masonry has led engineers to seek for some substitute. Such, for example, is the pneumatic pile of Dr. Potts, a series of cast iron cylinders filled with concrete or rubble masonry. Another example is Cushing's hollow pile of iron filled with wooden piles. Another, in which a very slender support is necessary to save space, is the iron pillar of various forms supporting the girders of the elevated railroads of New York. These pillars have to resist not only vertical loads, but considerable side thrusts. They are inserted in cast iron sockets, which are bolted to foundations of brickwork, so that they are beams in fact, fixed at one end and loaded at the other, the loads causing both longitudinal thrust and transverse strains. All iron or wooden piers are open to the objection of the perish-

able nature of the material, exposed as it is to the action of the elements. The forms used do not always afford sufficient strength for permanency, except by a free use of material which might appear extravagant. There can be no question but that stone is the most desirable material for bridge piers in any situation, provided that the size and cost of a pier can be brought within the necessary limits in a given case. It is therefore a pertinent inquiry how may the size and cost of masonry piers be reduced without a sacrifice of their stability. To answer this question is the object of this paper.

In the first place the size of a pier may not be reduced in any considerable degree by improving the quality of cement used. For though the quality of cement should by no means be neglected, yet it is the universal judgment of engineers that whatever strength may be imparted to a structure by the cement should be omitted from the calculations of its stability, allowing the adhesion to furnish an additional and somewhat uncertain margin of safety.

In the second place the desired object is not to be attained by the use of dowels or metal clamps between the stones of adjacent courses. For, as these are usually inserted, they are inert, nor can they offer any tensile resistance until they have been somewhat stretched by the incipient opening of the joints which they are designed to protect. They are therefore effective only when the structure is on the point of rupture. Their use presupposes a state of tension in a portion of the bed joints which is not compatible with a true stability, for, if it were, then might the adhesion of the cement be taken into account also.

In the third place, no advantage is to be gained by any system of dovetailing between the blocks, since the dovetails are only effective against tensile forces, which should not be permitted in stone structures. Moreover, as in the case of clamps, the adhesion of good cement affords more resistance than the dovetails. Mr. John F. Bourne states in his report on the construction of the Roman Rock Lighthouse, at the Cape of Good Hope (Proc. Inst. C. E., Nov. 1868): "When well made, the Portland cement soon becomes as hard and tenacious as the stone itself (granite), entirely dispensing with the necessity of dovetailing, which could only be of temporary service."

Finally, a monolithic structure is out of the question, owing to the labor and expense of securing and transporting stone shafts of the necessary magnitude, and the difficulty of attaching them so rigidly to

their foundations as to permit the transverse strength of the stone to be developed. Aside from these objections, it is doubtful if many engineers would be willing to risk an expensive superstructure upon the transverse strength of any stone support. Whenever a pillar has been required to withstand a considerable transverse strain, it has been made of some other material than stone.

In general, it may be said that no dependence is to be placed on any method which develops either a tensile or transverse strain in stone; and it may be formulated as a maxim that no stone structure should be subjected to any lateral strain which may not be resisted by the pressure on the beds of the several courses, producing friction as against sliding, and a moment of stability as against overturning. This being granted, it follows that, if by any means the normal pressure on the beds can be increased, the stability of the structure will be augmented; or conversely, if in a structure of given stability, we can supplement the force of gravity by some other available force producing vertical pressure, we may reduce the given amount of masonry without impairing the stability.

Suppose, then, that we add to the pressure due to gravity a pressure due to the elastic tension of a steel rod which has been placed vertically in the masonry, and stretched nearly to the elastic limit. Let the rod be anchored in the foundation, passed up through the core of the masonry, and put into a state of tension by a nut at the top. Its tensile force is now exerted upon a washer, which in turn presses upon the masonry with an equal force, and the pressure is transmitted through every joint to the foundation. We now have the benefit, not only of the attraction of gravitation between the earth and the stone blocks, but also of the molecular attraction between the particles of the steel rod, both forces acting to produce stability, and both being permanent in their nature, while the proportion of parts may be such that the pressure due to molecular attraction may be far the greater of the two. Observe that a rod in the condition described is neither a dowel to prevent sliding, nor an ordinary clamp to limit the extent of motion after rupture of the joints begins—it is no mere passive resistant. On the contrary, it is the source of a perpetual static pressure, acting upon the pier in which it is placed precisely as does the attraction of gravitation (since it passes through the centre of gravity), but with the additional advantage that its point of application is at the top of the pier, pressing the top stone

equally with the bottom one, thereby greatly increasing the stability of the upper portion, and rendering all clamps and similar contrivances unnecessary.

With such a pressure as this at command we may proceed with confidence to diminish the size of the structure, leaving only stone enough to resist safely the maximum compression to which it may be subjected, and breadth of base enough to afford the necessary lever arm. The ordinary pier thus is reduced to a column, or a pair of columns, one under each truss, with a very remarkable saving in the quantity of masonry, without sacrifice of stability. But the saving in quantity of stone and, consequently, in cost, is not the only benefit derived from this mode of construction. There are several other advantages incidentally growing out of it. The amount of stone is so limited that we can readily afford to use the best quality, and to expend a large amount of labor upon it, also to use the best quality of cement and of any desired richness, thus producing a structure of the very first class, where, according to the ordinary method, only rubble work could have been afforded.

Again, the dimensions of a column in plan will in most instances be within the practicable sizes of single stone, enabling us to dispense with vertical joints, and so adding materially to the strength and permanence of the structure. Vicat concludes, after a variety of experiments, that the division of a column into courses, each of which is a monolith with carefully dressed joints and properly bedded in mortar, does not sensibly diminish its resistance to crushing, but that this does not hold good when the courses are divided by vertical joints. Again, a structure of cubic blocks forcibly bound together by a central rod under tensile strain cannot be shaken to pieces by the jar of passing loads, as the top courses of ordinary piers are liable to be, unless well clamped. Again, the small diameter of the column occupies much less space than the ordinary pier, often a desideratum in a water way or crowded thoroughfare, so that by this method we are enabled to use stone, where otherwise it would be out of the question. Columns reinforced by steel may safely be carried to a height of 12 or 15 diameters, beyond which they are subject to the law governing all long columns, and may fail by flexure rather than by rupture at the base.

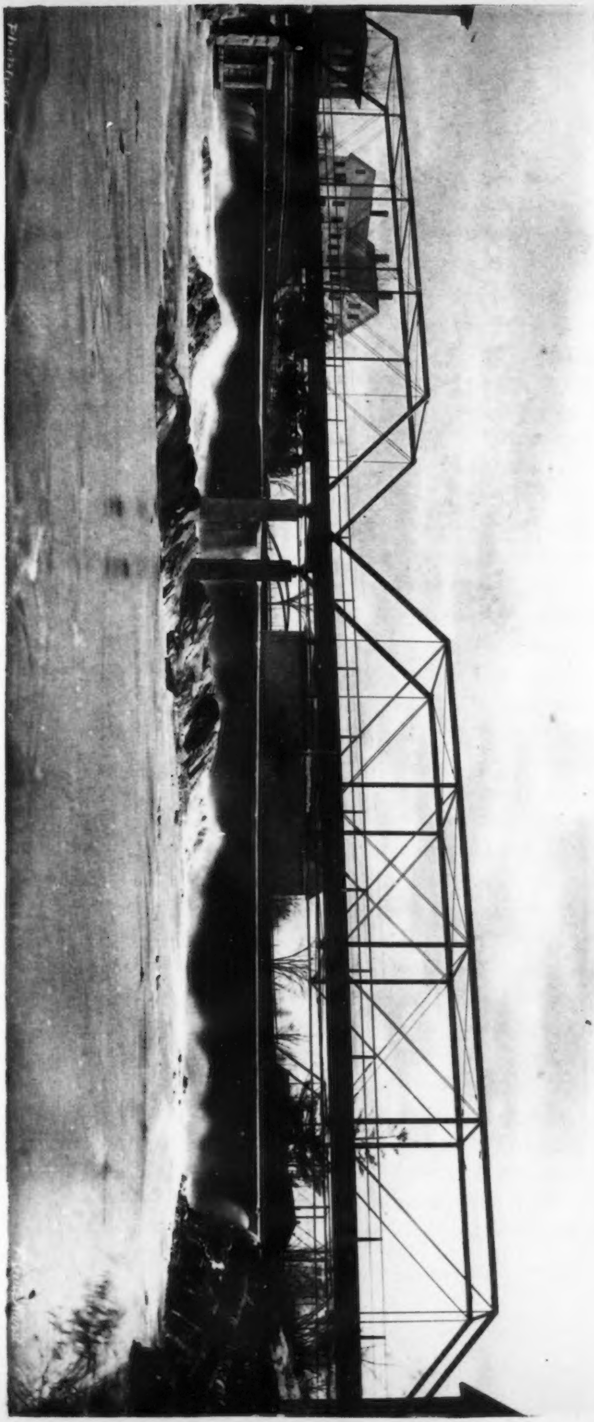
The use of metal to fortify masonry is by no means without precedent, although the manner of application has been different. Tie-bars

have been used to receive the thrust of stone arches, as in the Milan cathedral, and some other Gothic buildings.* Bands or chains of iron take the thrust of massive domes, as at St. Peter's in Rome and St. Paul's in London. Smeaton placed chains around the Eddystone tower, to receive the thrusts of the arched floors ; and Mr. Douglass informs us that the upper part of this structure has been strengthened on two occasions, viz., in 1839 and 1865, with strong internal wrought-iron ties, extending from the lantern floor downwards to the solid portion of the tower. Although previously the joints of the masonry in the tower had frequently yielded to the heavy strains imposed on them, and the sea water had been driven through them to the interior of the building, yet since making those repairs no further serious leakage has occurred, and the tower is now in a fair state of efficiency. In all these instances, although the metal appears to have been used merely to *resist* such forces as might come upon it *through the masonry*, yet they are useful as examples illustrating the permanence of masonry depending on the tensile strength of iron for its stability.

The principle of utilizing the elasticity of metal to produce *an applied force* for increasing the stability of masonry, as described in this paper, appears to be a novel one. A description of some columns erected by the author in 1878, upon this principle, may be of interest in this connection.

It was desired to erect an iron bridge in two spans of 97½ feet each, across the Presumpscot river, at Cumberland Mills, Maine. (See Plate XLVII.) This stream, which is the outlet of Sebago Lake, is very rapid, and is utilized at several points for water power. It is subject to sudden and violent freshets. The site of the bridge is just below the dam of the Cumberland Mills Company, and upon rapids descending at the rate of about 2 feet in 100. The bottom, at this place, is a mass of ledge rock, which, fortunately, in time of low water, forms a small island at the site of the pier. This enabled the work of preparing the foundation to proceed without interruption from the water. The pier consists of two columns, one under each line of truss, each 2 feet square, and 14 feet high, and entirely independent of each other. A vertical section of one of the columns is shown (Fig. 1, Plate XLVIII). Each foundation was prepared by cutting a place 2 feet square in the rock, and dressing it to a fine and level bed. At the centre of the square, a hole 4 inches in diameter and 30 inches

* Cadell's Journey through Carniola and Italy. Vol. II, p. 77.



deep, was sunk with a Wood's steam drill. The hole was enlarged at the bottom to about 6 inches on one diameter, giving an oval section. The enlargement was confined to the lower 10 inches, and was done with hand drills shaped purposely for this work. The hand drill was kept up to its place by a cast iron cylindrical rest, 3¾ inches in diameter, which was dropped down one inch at a time, as the work of enlarging proceeded. The hole was then calipered at every inch in height, and a wrought iron wedge 14 inches long was forged, of such size and shape, that it would spread the rod so as to exactly fill the hole. The rod was of Bessemer steel, 3 inches diameter, and divided at the lower end by a longitudinal channel, cut through it, 3-16ths wide, and 18 inches long. About a quart of thin paste of clear Portland cement was poured into the hole in the rock, and the rod immediately lowered into it, with the point of the wedge sticking in the jaws. The rod was then driven home over the wedge, with a 450 lb. hammer, having a five foot fall. The hammer was a piece of annular cast iron, sliding loosely on the rod. Its blow was received on two oaken blocks firmly clamped to the rod by four bolts, About twelve blows were required to drive the rod to the bottom of the hole. The upper end of the rod, on which a screw thread had been cut, was meantime held in its true position by a notch in a plank attached to the scaffolding. The remaining space in the rock about the rod was filled with a mixture of Portland cement, iron filings and sal-ammoniac, with water. This was allowed to set four days before laying stone. The stone blocks were of granite, taken from the Yarmouth quarries, near Portland. They were from 18 to 28 inches in thickness, and were finely dressed on two beds so as to make a joint not exceeding one-eighth of an inch. The sides were left with the natural or rock face, only the corners being pitched off to a line. Each block was perforated with a 4 inch hole, by the same rock drill. When the hole was half through, the block was turned over, and the hole finished from the centre of the opposite bed. Each stone, after being carefully cleansed with water, was lowered down over the rod, and set in a thin paste of Portland cement and sharp sand in equal parts. The annular space between the rod and stone was filled with perfectly dry cement powder, the object of this being to protect the steel from rust, without permitting any adhesion between the rod and stone. Upon the top stone was placed a cast iron washer, 9½ by 14 inches, having a central boss, the upper surface of which was turned down smooth. The washer was laid in cement, and a steel nut was

screwed down to a bearing. The nut was then turned still further, so as to stretch the rod twelve-hundredths of an inch. The free length of rod being 14 feet, if we assume the co-efficient of elasticity to be 28 000 000, the strain in the rod is $\frac{0.12 \times 28\,000\,000}{12 \times 14} = 20\,000$ lbs. per square inch. A small set screw was then placed in one side of the nut to prevent the possibility of its ever turning, and the column was complete. A third column was required at one end of the bridge, but otherwise the bridge was supported on some foundation walls of adjacent buildings. The total quantity of stone in the three columns is less than 6 cubic yards.

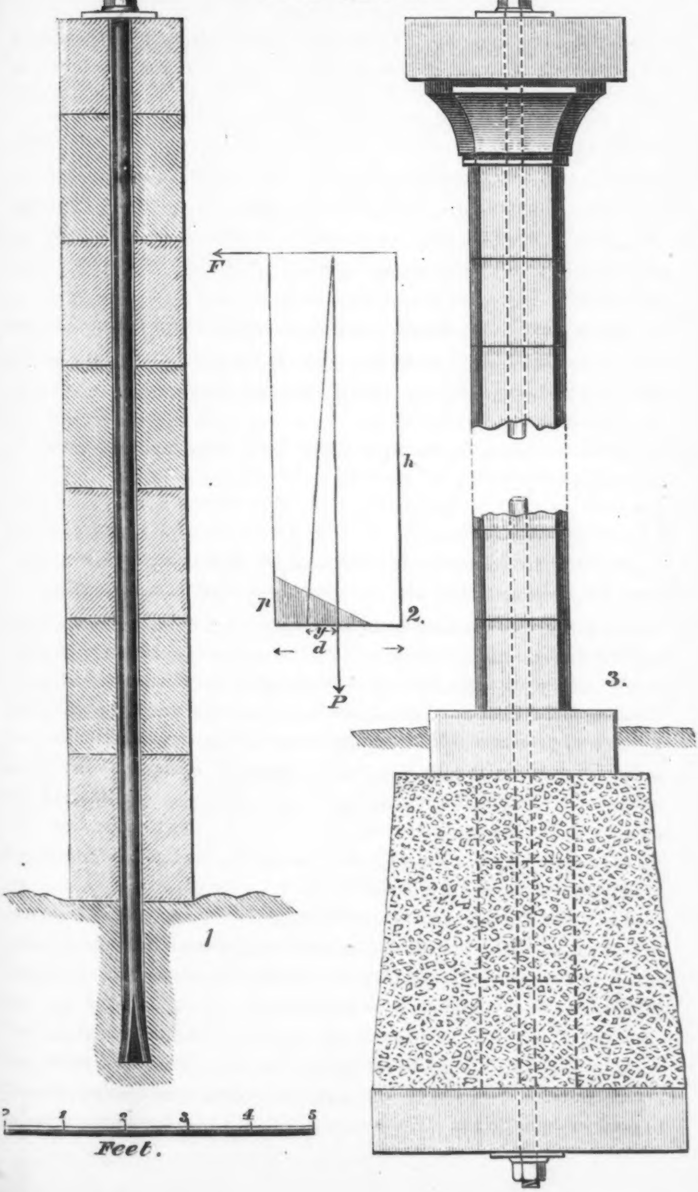
The stability of a column built in this manner is calculated in the same way as for crdinary masonry. We have the weight of the stone and of the load on the column, combined with the pressure due to the tension on the rod, forming a vertical force, which, multiplied into a suitable lever arm in the base, produces a moment of stability; and this must sufficiently exceed the moment of the extraneous forces. But on account of the limited area of the beds it becomes necessary to consider also the compressive strength of granite. Gen. Gillmore gives, as the result of his experiments of fifty-five samples of granite in 2-inch cubes from various sources, for the

Least crushing strength per square inch, on bed,			9 450 lbs.
Greatest	"	"	24 040 "
Average	"	(55 specimens)	16 423 "

He also shows that the strength per square inch is much greater in cubes of larger size. Probably the average crushing strength of large blocks used in a column would not be less than 20 000 lbs. per square inch of bed, one-tenth of which may be used as a safe compressive strength.

If a column (Fig. 2, Plate XLVIII) of height h is subjected to a horizontal force, F, at the top, the moment at the base is Fh, and for equilibrium with the vertical pressure, P, we have $Fh = Py$, in which y is a variable lever arm measured from the centre of the base to the centre of resultant pressure, and varying directly as F. In the calculation the weight of the column is assumed to be applied at the top, and the assumption is on the side of safety. The maximum value which may safely be allowed to the lever arm, y, will determine the maximum force, F, which the column may safely resist. In ordinary masonry of good quality, subject to side pressure, the practice is to allow to y a maximum value ranging from $\frac{1}{4}$

Plate XLVIII.



to $\frac{3}{8}$ths the diameter of base. We may assume, therefore, $y = \frac{d}{4}$ for a maximum safe value in a column without vertical joints, and reinforced with steel, provided that this position of the resultant does not induce an unsafe pressure at the edge of the joint. In some respects this column resembles an arch of stone voussoirs, in which, although the principle of restricting the resultant to the "middle third" is a good one, yet practice justifies allowing to it the latitude of the middle half, provided that the pressure on the intrados or extrados is not thereby carried beyond safe limits.

When $y = \frac{d}{4}$, the diagram of vertical pressure is a triangle extending over three-fourths of the diameter, its area is equal to the vertical pressure P, and its perpendicular p, represents the pressure per square inch at the surface of the column. If we call b the thickness of a rectangular column at the base at right angles to the diameter d, the value of p is expressed by

$$p = \frac{8}{3} \cdot \frac{P}{b\,d}$$

According to this *theory*, one-fourth of the base is relieved from pressure, but there can be no tension on that part since the pressure on the other part furnishes a moment of resistance equal to the moment of the applied forces. Since, however, *in fact* there must be tension on the one-fourth (unless the stone be absolutely inelastic), the tension, if within the resisting strength of the cement will form a small moment, by which the moment of pressures will be reduced; that is to say, the value of the actual lever arm y, is a little less than shown by theory. If the joints were laid dry, without cement, they could only open one-third of the amount of the compression of the adjacent blocks due to a pressure per square inch equal to one-fourth p as found above. So that the "opening" under any safe force F is practically *nil.*

The strain developed in the rod through the nut, in the first instance, is neither increased nor diminished by any external transverse forces which do not exceed the maximum for which the column has been proportioned. In the equation of moments $Py = Fh$, y and F are the variables, and will so continue until y is limited by the size of the column, that is, until $y = \frac{1}{2}d$, when rupture will take place. But under the maximum value of F, y equals only $\frac{1}{4}d$, which requires no change in P to produce equilibrium. The strain of the rod may be supposed to be

replaced by the weight of a heavy body on top of the column, producing the same pressure, without altering the conditions of the problem. The strain in the rod can only be increased by further stretching the metal, and this can only be done by either expanding the stone or opening the joints, *neither* of which can be effected by the maximum safe force F. Similarly the strain in the rod can only be diminished by a compression of stone along the axis of the column, which the maximum safe force F cannot do. We are therefore not concerned about a factor of safety in the rod, but may use its tenacity up to the so-called limit of elasticity. Indeed, there appears to be no good reason for not carrying the strain still higher, for although the permanent set would tend to diminish the effective extension of the metal and consequently its tensile resistance, yet within reasonable limits of strain there would be on the whole a gain in efficiency.

Since the permanency of the strain in the rod depends on the unyielding nature of the stone, it is important that the joints be laid in cement, which, as it hardens, resembles stone in its qualities. The use of lead in the joints cannot be allowed, because of the tendency of that metal to "flow" under strain, and so gradually to shorten the column.

It is not necessary to the system that the foundations should be on native rock. An artificial foundation may be prepared in earth or other material, the only essential thing being a continuity of solid stone between the bearing points of the rod, to insure permanency of strain. Fig. 3, Plate XLVIII.

The stability of the column described is found as follows :

Stone, 56 cubic feet × 165 pounds =	9 240	pounds.
Bridge, 97.5 feet × ½ × 800 " =	39 000	"
Steel, 7.07 sq. in. × 20 000 " =	141 400	"
Total....................	189 640	"
With ½ foot lever arm, moment of stability = 94 820 ft. lbs.		
Live load 97.5 feet × ½ × 2 000 lbs.	97 500	"
Total....................	287 140	"

With ½ foot lever arm, moment of stability = 143 570 ft. lbs.

These are taken as the maximum safe moments, the factor of safety being at least 2.

The maximum pressure per square inch on any part of the granite under these moments is 878 pounds, and 1 329 pounds, respectively, by the formula for p given above.

The assailing forces are : *First*, that of the water. Two of the most violent and destructive freshets ever known in that stream have occurred since the columns were erected. The greatest depth of water on the column was 8 feet. This on an inclined bed of 2 feet per 100 gives a calculated velocity of 14.9 feet per second, and a pressure upon a vertical surface at right angles to the current of 724 pounds per square foot. As the column is set with a diagonal parallel to the current, the other diagonal, or 2.828 feet, is the width of column opposed to the stream, but since the faces stand at an angle of 45 degrees, the *effective* width is 68 per cent. of this, or 1.923 feet. This, multiplied by 8 and by 724, gives the total pressure on the column of 11 138 pounds; and this by a lever arm of 4 feet (half the depth) gives a moment of 44 552 feet pounds, which is less than half of the above moment of stability. The factor of safety is consequently over 4. Should the water be 10 feet deep, the calculated moment of pressure would be 83 130 feet pounds, and factor of safety would be over 2.

Second, The impulse from a suddenly stopped train on the bridge.—Estimating the train weight on one span at 97.5 tons, half of which comes on one column, and the brake force at 200 pounds per ton, we have a total horizontal pressure of 9 750 pounds applied at the top of the column. The height being 14 feet, produces a moment of 136 500 foot-pounds, and the factor of safety is over 2 in this extreme case.

The slight and elegant appearance of such columns, when their strength is made evident, renders them particularly appropriate to the support of the modern truss bridge, which is so airy a structure as to make a pier of mass masonry look still more cumbersome by contrast. The principle is applicable, however, to masonry designed for other purposes than piers, such as lighthouses, breakwaters, dams, retaining walls, etc. It is proper to state, in conclusion, that this method of construction is secured to the inventor, Charles E. Hill of New York, by letters patent in Great Britain, Canada, and the United States.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

(Vol. VIII.—September, 1879.)

DISCUSSION ON PAPER CLXXXV.

STABILITY OF STONE STRUCTURES.

BY O. CHANUTE, CHARLES MACDONALD, D. J. WHITTEMORE, THEODORE COOPER, R. HERING, CHARLES E. EMERY, and WILLIAM H. SEARLES.

DISCUSSION SEPTEMBER 3D, 1879.

O. CHANUTE.—Granting that all the calculations of strength and stress are correct; granting that the construction was perfectly done, and that in the case described in the paper the rod was not drawn up into the dovetailed hole in the rock at the bottom, the $\frac{12}{100}$ (less than $\frac{1}{8}$) of an inch, which the nut was screwed down at the top, the effect of which would be to destroy the initial tension which is estimated by the author at 20 000 pounds per square inch; granting that the tension put upon the rock at the base has not cracked it, and so relieved the rod; granting also that changes of temperature have no effect upon the rod, while we know that if it were in the open air it would be exposed to variations of 150 degrees, which would expand a rod 14 feet long $\frac{15}{100}$ of an inch—it yet seems difficult to see any extended application for this ingenious method of saving masonry.

The instances where solid rock is found at or above the surface of the water, just where we want to locate bridge piers, are, as we know, exceedingly rare. The rock, if rock there be, is generally overlaid with some other material.

Even if this be only water, it would be difficult to make a good job of dressing off the bed for the pier, and drilling the dovetailed hole, say at a depth of 10 feet, especially if there should be a current, and there would be further trouble in stringing the stones accurately upon the rod, after it was anchored.

In fact, the difficulty seems to be to get the composite column into place so as to render it efficient, as this involves making it fast to something at the bottom. We cannot drive it down through the mud like a pile ; we cannot well sink it through sand like a pipe, by a jet of water, as was done by Mr. Macdonald at Coney Island ; and we cannot excavate inside of it, as in a cylinder.

We can, to be sure, put down a coffer dam or a caisson through the overlying material, and, if not too deep, pump it out, and fit the column to the rock ; but in that case the saving in masonry is likely to be more than made up by the foundation expenses. Or we may drive a pile platform under water, cover it with a grillage, and set the column upon that ; but it will then be difficult to connect the column efficiently with the foundation, and the stability against a side blow is likely to be that due to the friction on the bottom—due to the weight alone.

This friction, if the column rests upon stone, will be 60 per cent. of the incumbent weight. If it rests on timber it will be 40 per cent. Now, in the case described, the weight of one column, and of the portion of the bridge which it carries, is stated to be 48 240 pounds ; so that if it rested upon the rock, instead of being anchored to it, its stability against a side blow would be equal to 28 944 pounds, while the thrust of 8 feet of water is estimated by the author at 44 552 pounds. If it rested on timber the stability due to the friction at bottom would only be 19 296 pounds.

Charles Macdonald.—It is difficult to see how the system proposed can be carried out in practice with economy.

If the only object of the steel rod is to "supplement gravity," the same result could be obtained at less expense by simply adding dead weight upon the top of the column, in which case the practical questions referred to in the discussion, as resistance to impact, pressure of ice or floating timber would still have to be met. These and other considera-

tions would probably put this system outside of what may be called staple methods of construction.

D. J. Whittemore.—The stability of masonry of this kind depends on the strain imparted by the steel rod remaining always constant. This I cannot conceive to be practically possible. With a variation of temperature of say 90° Fahr. the unequal dilation of the two materials, granite and steel, would materially change the value of stress imparted by the steel rod.

I cannot conceive that the anchorage into the bed rock in the manner specified to be such as would resist the desired strain for any considerable length of time. In my opinion, the vibration imparted to the columns by winds upon, and traffic over the structure, would in a short time cause such action at the anchorage as to permit the rod to move the $\frac{12}{100}$ of an inch, thereby relieving the pressure intended to be exerted by the rod.

It would be interesting to know whether more than the ordinary appliances were required to turn the nut at the top of the three inch rod to procure the alleged strain of 141 300 lbs.

Theodore Cooper.—The columns described in this paper may be applicable in particular localities, but that they should ever be able to replace ordinary pier construction to any great extent, appears very doubtful.

The author states that the rods in the columns under the bridge over Presumpscot river, were given an initial strain of about 20 000 lbs. per square inch, but does not state whether this was done after the bridge was fully loaded or not. If done previously to the imposition of the weight, the rods would have little or no strain, upon the passage of a load, when it needs its stability against the forces due to this train. For he states the total strain on the column from the rod to be 141 300 lbs. and the strain due to the load (dead and live) to be 136 500 lbs.

In his consideration of the "assailing forces," he considers the pressure of the water against the surface of the column only—and gives no consideration to floes of ice, rafts, boats and floating flood trash; few American rivers or streams are free from some of these, and we think no structure can be considered stable which is not prepared to resist the blows or pressures due to such bodies.

He finds a factor of safety for these particular columns of 4 and 2 for 8 and 10 feet of water respectively—with a surface exposed to pressure of 1.925 feet × by the depth of water.

It would not require much ice or floating logs or timbers to increase this surface sufficiently to reduce his factor of safety to nothing.

If ordinary substructures in running streams were calculated upon the same data as the author has adopted, their dimensions might be reduced to very small ones, but few engineers would like to venture upon such proportions.

For locations free from water, it is very questionable, unless the proper data for calculating the cost of work upon these columns is furnished, if iron columns would not be far cheaper and better adapted for the purpose, especially when the facility for bracing cheaply is so easily obtained.

R. HERING.—This paper is very interesting, and I believe the novel idea illustrated may be applied with advantage in certain cases.

Fearing that the author may be subject to some misinterpretation, I would like to emphasize an essential point which, as I think, has not been made sufficiently clear.

He says that the tensile strain of steel rods compressing the structure may "supplement the force of gravity."

This is only conditionally true, and upon the proper disposition of this strain will depend entirely the success or failure of the pier.

The moment of stability is assumed to be the weight of the stone plus the pressure due to the tension of the rod multiplied into a lever arm equal to one-fourth of the base. This can only be true, if the pressure due to the rod is an *applied* or *motive* force, acting like gravity. What, however, is the condition of the strain in the rod? There is a force acting upwards against the direction of gravity of precisely the same intensity as the one acting downwards, as in the case of a block which is simply compressed by the action of such rods, but which, when set upon the ground, would certainly not have its stability against overturning increased thereby,

An essential consideration must be observed, which is, that, in order to obtain an applied or motive force, such as is brought into the calculation, it will be necessary to neutralize the force acting upwards. This can only be done by anchoring the rod in the foundation to such an extent that this entire upward pull will be safely resisted by it, *i. e.*, neutralized and prevented from acting in opposition to the downward force which we desire to utilize.

An anchorage furnishing such resistance is indispensable to the success of the plan, and it seems to me that the author has not sufficiently insisted upon this, and therefore might cause misapprehension. The only reference is made, when he says, " let the rod be anchored in the foundation," but to what extent this must be done, is not mentioned.

In Fig. 1, Plate XLVIII, he describes an executed example, and says that a hole was drilled into the rock 30 inches deep to anchor the rod, but nothing is said about the capability of the rock to withstand a lifting tendency 141 300 lbs. (of course before the weight of the column is on it), although this fact is essential to the stability of the described piers, when subjected to the extraneous forces which he assumes.

A further remark might also tend to mislead the reader on this point, when Mr. Searles says that heretofore metal was "used to merely *resist* forces coming through the masonry," but that the present novel case is (without giving the conditions) a "principle of utilizing the elasticity of metal to produce an *applied force* for increasing the stability of masonry."

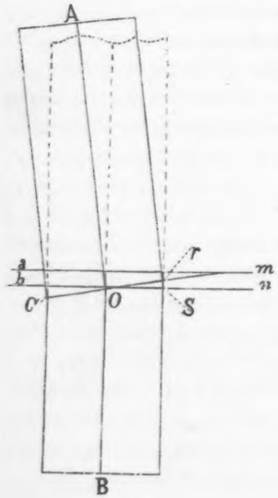
Nor does he allude to the importance of this question, when he says that "it is not necessary to the system that the foundations should be a native rock," but that "an artificial foundation may be prepared in earth or other material," and adding, "the only essential thing being a continuity of solid stone between the bearing points of the rod."

It is questionable in my mind whether this method will be economical in any case, except when the foundation is solid native rock, for the same amount of masonry which is saved in the upper part of the structure by utilizing the *downward* strain in the rod as a motive force would generally have to be built into it as a foundation or anchorage to get weight enough to resist the *upward* action of that same strain.

The only disadvantage I can see in utilizing the elasticity of a rod for the purpose recommended in the paper is the precision with which the true elongation must be ascertained when the rod is finally put under tension, as every hundredth part of an inch corresponds, in Mr. Searles' case, to over 1 600 lbs. To be certain of the exact amount of motion, when tightening up the rod, which is solely due to its extension but does not contain an element due to the compression of the masonry, requires a delicacy of adjustment which is difficult at best, and in many cases may perhaps be impossible.

Charles E. Emery—I have examined this paper with considerable interest, from the fact that it appears to be a practical application of the

principles involved in a discussion in which I was engaged some years since on the subject of initial tension.* I am pleased to state that this previous consideration of the general subject, in connection with the formula in my recent paper on "The Flexure and Transverse Resistance of Beams," enables me to suggest what I think will be accepted as a more complete analysis of the conditions of equilibrium of all columns than that offered in the paper under discussion, and giving results somewhat more favorable to stability.



In figure let $A B$ represent one of the columns, and suppose any section to be compressed by the dead load, and an internal rod from a plane in the line $a m$ to another in the line $b n$. If now a lateral force be applied at the top of the column, it has no tendency to increase or decrease the *total* vertical force, but, as in any case of transverse resistance, equal and opposite forces must be developed on opposite sides of the column to resist the lateral moment. Were the column homogeneous and not under vertical strain, the movement due to the moment, as well as the strains developed, would be represented by two triangles, $b o c$ and $r o s$, and the initial strain does not change this representation, for such strain is simply increased on one side by the forces represented by the triangle $b o c$, and decreased on the other by the forces represented by the triangle $r o s$, the moment being balanced by the difference in strain as before. So long as the forces produced by the moments on the side last above named are less than

* The discussion referred to was occasioned by an article in the "Railroad Gazette," to the effect that tightening the back braces of a bridge increased the strains on main members when supporting external loads. The contrary appeared so evident that I went further than necessary by asserting that it was a general principle that strains due to internal tension or by the tightening of one member upon another were not increased by the application of external loads producing strains of less intensity. This appeared accurate at first sight, from the following illustration: If a spring balance were hung up, and maintained in a state of tension of say 50 lbs. per index, by struts between cross pieces in the rings, evidently any load less than 50 lbs. would not loosen the struts, and hence the strain on the spring would

those due to the initial tension, evidently the strain on the rod will not be altered. The whole structure, including the rod and stone, is, under such circumstances, in the condition of the compressed side of a beam. Before the lateral moment acts consecutive sections of the beam are parallel, and may be represented by the lines am and bn, but on the application of the bending moment such lines converge, and the axis for reference corresponding to the neutral axis, rapidly approaches the column, but should in general remain outside. The lateral force may, however, be increased until the neutral axis moves within the section of the column, in which case the vertical lifting force due to the moment has overbalanced the downward pressure, and strain is thrown upon the rod in addition to its initial tension. When, however, the strains from the moment are kept less than the original tension, the former may be separately considered. In my paper referred to,* §43, it is shown that the centre of effort of the forces either side the neutral axis for a rectangular section are situated at distances r_0 and r_1, from the axis equal respectively for a homogeneous material to $\frac{2}{3}$ the semi-diameter. Using my notation of $M=$ the bending moment and N the longitudinal force due thereto, which acts in opposite directions on the two sides of the axis, we have for my paper (Eq. 20) $M = N(r^0 + r_1)$, or using Mr. Searles' notation as far as applicable

$$(1)\quad Fh = \frac{2}{3}\,Nd$$

in which $F=$ the lateral force at the top of the column, h the hight, and d the diameter of the column. Hence

$$(2)\quad N = \frac{3\,Fh}{2d}$$

If P represent the total vertical stress derived from the dead weight and the tension on the rod, as explained previously, the outer joint will not open, or the tension on the rod be increased, so long as $N < \frac{P}{2}$, and

---

not be increased until the external load produced strains greater than the internal tension of 50 lbs. Mr. J. F. Flagg, however, called attention to the fact that this would be true precisely only when the struts were inelastic, when Mr. E. S. Philbrick pointed out that the reacting elasticity would in general soon exhaust itself, the sections of struts being usually larger than those of ties, and that, therefore, the strains would only be increased slightly by external loads. All these suggestions together showed the true solution of the question; but it required a mathematical calculation to ascertain exact values which I expected to present before this, and which will show the original statement practically true, viz., that tightening the back braces of a bridge as constructed (in practice) does not increase the strain on main members under external loads, though the proposition, as a general principle, requires the modifications referred to.

* Flexure and Transverse Resistance of Materials. CLXXXI. Vol. VIII. June. 1879.

for this condition we have from equations (10) and (17) of my paper for the unit strain due to the lateral moment at the edge of a square column,

$$(3)\ f = \frac{4\ N}{d^2} = \frac{6\ F h}{d^3}$$

but as this is in addition to the initial strain on one side of the column, we have for the maximum unit strain

$$(4)\ f^1 = \frac{4\ N}{d^2} + \frac{P}{2d^2} = \frac{8\ N + P}{2d^2}$$

For the most extreme example given by Mr. Searles, viz., a force $F =$ 9 750 lbs. acting at a height, $h = 14$ feet with diameter, $d = 2$ feet, we have from (2)

$$(5)\ N = \frac{3 \times 9\ 750 \times 14}{4} = 102\ 375 \text{ lbs.}$$

$$(6)\ \frac{P}{2} = 143\ 570 \text{ lbs.}$$

Hence in this extreme case $N < \frac{P}{2}$ so the joints will not be opened or the strains on the rod increased. From (4) the maximum strain per square inch of stone under above conditions is $f^1 = 960$ lbs.

When the strains are applied in the direction of the diagonal of the section, as would in general be the case in the example given, the distance between the centres of effort $(r_0 + r_1) = .707\ d$, so the several equations above take the following forms:

$$(2a)\ N = 1.414\ \frac{F h}{d}$$

$$(3a)\ f = \frac{6\ N}{d^2} = \frac{8.484\ F h}{d^3}$$

$$(4a)\ f^1 = \frac{6\ N}{d^2} + \frac{P}{2\ d^2} = \frac{12\ N + P}{2\ d^2}$$

Hence in the particular case $N = 96\ 506$ lbs.;

$f^1 = 1\ 254$ lbs.

That is the stability is greater than before, but the maximum strain per unit of area of the stone is also greater, though within safe limits.

In any case as soon as N becomes greater than $\frac{P}{2}$ increased strains are thrown on the rod and the factor of safety is limited by its ultimate

tensile resistance or the ultimate crushing resistance of the stone when the moment arm is reduced to the semi-diameter or a little less. It will be seen then, that so long as the lateral moment is not sufficient to overcome the initial tension the stability is greater than estimated though the ultimate resistance is as stated in the paper. These calculations are independent of the moduli of the material, but evidently depend upon the original adjustment and approximate maintenance of the initial tension under variations in temperature and other practical conditions mentioned by the other speakers.

The above equations are equally applicable when the stability is produced by dead weight only, and when the column is bent, forming an arch.

William H. Searles.—Inasmuch as the steel rod is enclosed in the granite, and since steel conducts heat some 20 times more rapidly than granite, the two will be of sensibly the same temperature at all times. If the column is heated to 120° Fah. on the sunny side, its temperature may be 80° on the opposite side, giving an average of 100° for the heart of the column, and for the steel. If the column was erected at a temperature of 70° we have only 30° of expansion to consider.

The expansion of steel, per degree, is..........	.00000600
The expansion of granite, per degree, is.......	.00000438
Difference is..........................	.00000162
and for 30°................................	.0000486

which corresponds to a change in strain of 1 360 pounds per square inch. Therefore, in the extreme heat the rod will be relaxed .068 of its initial strain of 20 000 pounds per square inch, or .045 of an initial strain of 30 000 pounds which is now adopted. For extreme cold the tension is increased, and the stability likewise, without, however, endangering the steel by undue tension.

The coefficient of elasticity of granite is probably about one-sixth that of steel, while the section of the stone in the example cited is 80 times that of the steel; hence under equal strains the compression of the granite will be .075 the extension of the rod. If afterwards a load is imposed, equal to the strain in the rod it will not relieve the rod of more than .075 of its strain. As the live load and horizontal force were both arbitrarily assumed for illustration, this refinement of correction was

omitted. It is of importance, however, when calculations are to be made upon exact data.

The parties for whom the columns described were built, were positive that the chances of impact by any heavy floating bodies were exceedingly remote; consequently no special provision against impact was made in this case. So, too, the depth of eight feet of water is the absolute maximum—the depth of ten feet is imaginary. We may not be able to decide from theoretical considerations what amount of impact such columns would safely resist; but it could hardly be less than that of any iron columns that might be adopted in their place. A granite column is now building at Mott Haven, N, Y., which is to be tested for impact as well as for other kinds of strain. It is hoped that these experiments will throw considerable light on this subject.

It ought not to need saying that the design of these columns, both in elevation and plan, can be varied *ad libitum* to meet the requirements of special cases. Thus, quite a heavy pier may be built up to the limit of the flood or ice line, to resist impact and extraordinary pressures from the strain, and yet be surmounted by a slender column of stone to carry the superstructure ; the whole being reinforced by a steel rod extending from coping to foundation, and affording, according to its size, any amount of stability desired.

It is, of course, obvious that when the rod is anchored in solid rock the rock must be able to resist the pull of the rod, otherwise the column in overturning would carry away a portion of the solid rock with it. The surface of a right cone 30 inches high, and having 30 inches base, is 1 580 square inches, among which the strain of 141 400 pounds being divided, gives 90 pounds per square inch as the strain on such an imaginary inverted cone in the bed rock when the rod is 30 inches deep.

In case of soft rock which might not resist the direct pressure of the rod, a larger piece of iron may be anchored securely in the rock to gain adequate resistance, and the rod screwed into this. A combination of nut and lewis pieces accomplishes the same purpose at any desired depth.

The stretch of the rod in fourteen feet would be 18 hundredths of an inch for a strain of 30 000 pounds, now adopted, and of course twice as much in a 28 foot column, so that the taller the structure the greater will be the security against a loss of tension in the rod.

When the bed rock is submerged or overlaid with other material much smaller coffer dams may be employed than for ordinary masonry,

and these may generally be left in place and filled with concrete around the columns, giving additional stability. Or a crib may be sunk to the rock filled with concrete, and a column complete in itself may be bedded in this material to a proper depth without going to the bed rock with the stone.

When the foundation is of sand, the columns may be built on a staging out of water, and afterwards sunk by the water jet process to any desired depth, the water-pipe passing down outside of the column and discharging at a central point beneath it, and afterwards being unscrewed and detached. Columns for this purpose may be 16 or 18 inches in diameter.

When the column is built upon earth, as in Fig. 3, Plate XLVIII, the foundation may be given any depth and bulk necessary to insure stability, while the granite core running through it to the bottom secures the resistance required for the steel, and enables the stone column to utilize more fully the stability of the foundation than an iron column generally can do.

Mr. Emery, in his discussion of strains, assumes the column to be a homogeneous beam, which it is not. While his formulæ seem to be applicable so long as the centre of pressure is within the middle third, yet inasmuch as we are mainly concerned with the maximum strains which may carry it beyond this limit, I prefer to consider the column as a voussoir arch, of infinite radius. The shaded triangle in Fig. 2 represents the reaction of the foundation at the lower joint. Its area represents the vertical force P, and its centre of gravity is over the point where the centre of pressure intersects the base. When $y = \frac{1}{4} d$, the base of the triangle $= \frac{3}{4} d$, and its area $= \frac{1}{2} p \times \frac{3}{4} d = P$ for a unit's thickness of column; whence, multiplying by the thickness b, we have

$$p = \frac{8}{3} \cdot \frac{P}{bd} \text{ as before,}$$

for the maximum unit strain in the granite at the surface of the column, when $y = \frac{1}{4} d$. Under these conditions the strain in the rod is not increased.

Most of the objections suggested will apply in a greater degree to iron columns, while the stone column has in its favor no limitation as to size, an increasing stability with increasing load, absolute durability of material, and no expense for painting or repairs. Lugs of metal may be inserted between stones during construction, for attaching braces,

&c., though braces are quite unnecessary, since the columns may easily be designed to be self-sustaining without them, and the trusses should be.

The question of cost is outside of a technical consideration of the principle involved; but since it has been raised it will be sufficient to reply that the cost of these columns is found to be not only less than that of ordinary masonry, but also within the cost of iron supports of equal strength when their necessary foundations are included in the estimate.

ERRATA.

Paper CLXXXIII. Transactions, Vol. VIII. August, 1879. Schmidt on "The South Pass Jetties."

Page 190. Third line from bottom, for 1879, read 1878.

" 204. Eleventh line from bottom, for *slope*, read *slopes*.

" 206. Top line, omit the word *long*.

" 206. Bottom line, for *cone*, read *core*.

" 208. Eleventh line from bottom, after *inch boards*, insert *as a flooring*.

" 219. Seventh line from top, for *when* read *where*.

" 223. Thirteenth line from top, for *dykes*, read *dyke*.